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Safety examination of existing concrete structures using the global resistance safety factor concept

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ABSTRACT

Current design procedures for structural concrete elements are based on *member level* (or local) safety checks, in the sense that the safety condition is evaluated at each cross-section individually. In the examination of existing structures, the exploitation of the redundancy of the structural system and its redistribution capacity may prove necessary to fulfil the safety requirements, demanding for a *system level* (or global) safety evaluation. This can be achieved using nonlinear finite element analysis (NLFEA) procedures and requires a safety format different than that based on partial safety factors.

In this work, a simple safety format tailored for NLFEA of existing structures is described. It is shown how to define a global resistance safety factor based on a simple semi-probabilistic approach in line with the recommendations of the new Swiss standards for existing structures [1,2] that can capture the sensitiveness of the structural system resistance, R, to the random variation of the input variables. Besides enabling the use of updated information regarding the material properties, the proposed procedure allows performing reliability differentiation based on risk analysis, being therefore suitable for the safety examination of existing bridges.

The definition of the semi-probabilistic global resistance safety factor is based on the assumption of a log-normal probability density function for the resistance R and on an estimate of its coefficient of variation, υ_R. The existing proposals for estimating υ_R are reviewed and compared. For statically determined structures with a single dominant failure mode (axial compression, bending or shear) the examination values of R computed with global resistance safety factors are shown to compare well with those obtained with partial safety factors. The reliability of the examination values of R is also evaluated through a comparison with the Monte Carlo simulation procedure and it is shown that the global resistance safety factor method is sufficiently accurate. Finally, a case-study is presented illustrating the application of the proposed procedure in the structural safety examination of an existing prestressed concrete bridge.

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1. Introduction

The examination of existing bridges constitutes a privileged field of application of detailed structural analysis methods such as nonlinear finite element analysis (NLFEA). Compared to design of new structures, more detailed structural analysis methods are justified in the examination stage since the economic impact of conservative calculations can be large and lead to unnecessary interventions. In general, a stepwise approach [3–5] is recommended in the structural safety evaluation culminating, if necessary, with a NLFEA [6]. Besides encouraging the use of engineering judgement throughout the condition evaluation, this procedure fits the Levels of Approximation approach [7] which is now also adopted in the *fib* Model Code 2010 [8].

However, it is well recognized [9–17] that the use of NLFEA in the structural safety evaluation demands for a tailored safety format, different than that based on partial safety factors. Current design and examination procedures for the ultimate limit state are based on the verification of the following inequality:

$$R_d \geq E_d \tag{1}$$

where E_d is the examination value (or design value, in the case of a new structure) of the inner force at the cross-section being analyzed and R_d is the corresponding examination (or design) value of the resistance. In the examination of existing structures, the degree of

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compliance n as defined in Eq. (2) can be used advantageously since it gives a measure of the relative safety margin [2]:

$$n = R_d/E_d \geq 1.0 \quad (2)$$

The examination values of the inner forces E_d are obtained considering the relevant load combinations and can be written as:

$$E_d = \gamma_G G_k + P + \gamma_Q \left(Q_{k,j} + \sum_{i \neq j} \psi_{0,i} Q_{k,i} \right) \quad (3)$$

where G_k and $Q_{k,i}$ are the characteristic values of the permanent and variable load effects, respectively, and P concerns the prestressing effects. The safety provisions consist in the amplification of the nominal action effects by appropriate partial safety factors γ_G and γ_Q. The combination factors $\psi_{0,i}$ reflect the unlikelihood of having the extreme values of several independent variable actions occurring simultaneously.

The examination resistance of a cross-section is typically determined in the form:

$$R_d = R(f_d, a_{nom}), \quad f_d = f_k/\gamma_M \quad (4)$$

in which the safety provision is applied at the material level by using the examination values of the material properties, f_d. These are obtained from the characteristic values f_k by reduction with partial safety factor γ_M, covering the random variability of the material properties. The geometrical properties are taken as nominal values a_{nom} and its variability is accounted for in the partial safety factors. While the inner forces are usually calculated using a linear elastic analysis, nonlinear behavior is taken into consideration in the determination of the resistance. Such a subdivision of the design/examination process results in an artificial, yet convenient, separation between the determination of the inner forces and the design/examination of the cross-sections.

The safety format outlined above is well suited for linear elastic analyses and local safety checks. In fact, the examination values of the material properties are extremely low and are not representative of a real material. However, as long as the action effects E_d are determined independently of the cross-sectional resistances R_d, the use of factored material properties does not interfere with the structural analysis. Moreover, as the inequality (1) is only evaluated at the cross-sections, this can be considered as a *member level* (or local) evaluation.

On the other hand, a nonlinear analysis constitutes a *system level* (or global) evaluation in which all structural parts interact leading to a generalization of the notions of resistance and action effect presented above. In general, system resistance is a function of the material parameters f, geometrical data a and loading pattern S:

$$R = R(f, a, S) \quad (5)$$

The loading pattern S encloses the notions of load type, location, combination and history.

If the reliability of the system (or global) resistance is to be evaluated, the effects of the random variation of the basic variables must be taken into account. In this case, the use of the partial safety factors in the determination of R_d is questionable. As remarked by several authors [9–11], any realistic simulation of structural behavior should be based on mean values for the material properties and the safety provision should be referred to it. Analysis based on extremely low material properties may result in an unrealistic redistribution of forces, which may change the failure mode [18] and may even not be on the conservative side. In this context, and still adopting the condition (1), the examination resistance can be obtained by division of the nominal resistance calculated using the mean values of the material properties f_m and nominal geometrical parameters a_{nom} by a global resistance safety factor γ_R:

$$R_d = \frac{R(f_m, a_{nom}, S)}{\gamma_R} \quad (6)$$

Concerning the design of new structures, a safety format consistent with the reasoning presented above is defined in the EN1992-2 [19], in which the definition of the global resistance safety factor is based on the work of König et al. [9] and is described in detail by Bertagnoli et al. [17]. More recently, several proposals for the determination of γ_R based on semi probabilistic approaches that can capture the system resistance sensitiveness to the random variation of the input variables were developed [11–16].

In this work a global resistance factor based on a semi probabilistic approach is adopted, following the guidelines of the new Swiss standards for existing structures [1,2]. Besides enabling the use of updated information regarding the material properties and the model accuracy, the proposed procedure allows performing reliability differentiation based on risk analysis and on the expected service life of the structure.

2. Safety format for examination of existing bridges based on nonlinear structural analysis

2.1. Basic reliability concepts

The safety requirements of a given structure can be brought into the form of a so called *limit state condition*, which can generally be written as:

$$G(\mathbf{X}) \geq 0 \quad (7)$$

In the expression above $G(\mathbf{X})$ is the limit state function and $\mathbf{X}$ the vector of random variables governing the problem. In the simplest case, the limit state function can be defined as the difference between the generalized structural resistance R and the generalized action effect E:

$$G(\mathbf{X}) = R - E \quad (8)$$

in which case G is equal to the safety margin. In a full probabilistic approach, the satisfaction of the safety requirement is expressed by the condition $p_f \leqslant p_0$, where p_0 is the target probability of failure and the probability of failure p_f, or probability of limit state violation, is defined as:

$$p_f = \text{Prob}[G(\mathbf{X}) < 0] \quad (9)$$

In general, the limit state function can be a function of many variables and a direct calculation of p_f is not possible. Stochastic simulation techniques, such as the Monte Carlo method, can be used to get a reliable estimate of the probability of failure. This method requires the knowledge of the joint distribution functions of all random variables, the evaluation of the limit state function for each realization of the random variable vector and a posterior statistical evaluation of the results.

2.1.1. Second Moment Reliability Methods

In most circumstances, alternative approximate methods are preferred instead, such as the Second Moment Reliability Methods. In these methods the distributions of the basic variables defining R and E are described solely by two properties, namely their expected values and their variances. In this context, it is convenient to measure structural safety in terms of a reliability index β, which is related to the probability of failure:

$$\beta = -\Phi^{-1}(p_f) \quad (10)$$

In the expression above, Φ^{-1} is the inverse of the standard normal probability distribution function.

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